G.W. Hunt

No. 522

REPORT

to the

Honorable Board of Directors

SANTA CLARA VALLEY WATER CONSERVATION DISTRICT

on the

TESTING LABORATORY

used on the

1934 Well Replenishment Project

PROJECT REPORT NO. 35

April 30, 1936.

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Office Of:
Chief Engineer
310 Sansome St.,
San Francisco

REPORT

on the

TESTING LABORATORY

of the

SANTA CLARA VALLEY WATER CONSERVATION DISTRICT

including

1) A DISCUSSION OF THE GENERAL REQUIREMENTS OF SOIL FOR ROLLED EARTH FILL DAMS
2) THE GENERAL PURPOSE OF THE LABORATORY
3) A DESCRIPTION OF THE LABORATORIES
4) A LIST OF EQUIPMENT USED IN THE LABORATORIES
5) A DESCRIPTION OF THE LABORATORY TESTS AND PROCEDURES
6) SUMMARY AND CONCLUSIONS OF TEST RESULTS

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Office Of:
Chief Engineer,
310 Sansome St.,
San Francisco

Subject: S.C.V.W.C.D.
TESTING LABORATORY
PROJECT REPORT NO. 35

April 30th, 1936

Honorable Board of Directors
Santa Clara Valley Water Conservation District
62 Grant Building,
San Jose, California.

Gentlemen:

The purpose of this report is to give a detailed description of the tests used and developed in the District's laboratory, with a general summary of the test results.

The scientific control of earth fill dam construction by laboratory and field tests has been developing rapidly within the last few years and the procedure is still undergoing change and standardization. It is thought that this detailed data may be of future help.

The text and graphs of this report were prepared jointly by Mr. C. B. Palmer, who had charge of the District's testing, and by Mr. G. C. Sullivan, principal assistant to Mr. Palmer.

Mr. Harold I. Wood, Supervising Field Engineer for the District, was responsible for establishment of the soil testing laboratory and procedure used.

Respectfully submitted,

Chief Engineer

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INTRODUCTION

In the latter part of 1934 and the early part of 1935, work was started on the six dams of the Santa Clara Valley Water Conservation District's well replenishment project. Four of these dams, known as Almaden, Calero, Guadalupe and Stevens Creek, are of the rolled earth fill type with a 4 inch reinforced concrete facing on the upstream face. The fill of these dams is divided into two sections, the upstream is of impervious material while the downstream is pervious, or less impervious than the upstream section. The impervious material is not strictly impervious, but the rate of flow through it is such as to be negligible.

The fifth dam, known as the Coyote dam, is of the rolled earth fill type, but it consists of only one section, impervious. Instead of a concrete facing, a rock facing is used. Each of the above five dams have a concrete lined spillway located to the side of the dam and a welded steel outlet pipe, incased in 9 inches of reinforced concrete, running through the dam for the passage of water.

The sixth dam, Vasona, has a rolled fill clay upstream section and downstream gravel fill section. The concrete spillway is located in the center of the dam and is controlled by radial gates. Concrete facing is also used on this dam. Fig. 1 gives the location of the dams and Fig. 2 the general size and capacity.

GENERAL REQUIREMENTS OF SOIL FOR ROLLED EARTH FILL DAMS.

In the design and construction of an earth fill dam, two main points must be kept in mind. These are, first the fill must be stable; and second, it must be of such a degree of imperviousness that seepage through the dam will not cause erosion or leakage; that is to say, excessive leakage.

By stability is meant the ability of the fill, foundations and abutments to resist sliding, swelling and heaving and settling. The resistance to sliding is determined by two factors, internal friction and cohesion. These factors depend upon the soil itself and the method of placing. The Bureau of Public Roads (9)[1] give methods of testing for internal friction and cohesion. In general, for high internal friction, the grading of the soil must be such that the individual particles will interlock with one another; thus preventing one particle from sliding over the other. Cohesion is obtained by the molecular attraction existing between the particles, and also due to the adhesion, skin friction and molecular attraction of a very thin water film between the particles. It is seen that hot only must the grading and structure of the particles be such as to allow the above forces to act, but the soil must be placed so that all of the particles are in intimate contact resulting in a low voidal space. To arrange the soil so that the particles are in intimate contact and are interlocked, they must be lubricated and then forced by external means

[1] See Bibliography

into a compact mass. The lubrication is obtained by the addition of moisture. The amount of moisture must be just sufficient to wet the particles and allow them to arrange themselves under the action of the compacting force, and yet be kept low enough so that the film of water surrounding the particles will not destroy the cohesion or prevent them from interlocking. The greater the compacting force, the less moisture needed for lubrication.

Swelling is due to capillary pressure acting on the particles causing expansion. Since capillary pressure is a function of pore width and is appreciable only when the pore width is below 0.005 m.m., swelling occurs only in silt, clays and colloids. In well graded soil; that is, soil containing sand and gravel besides the silt and clay in amounts just sufficient to act as a binder, capillary pressure is broken by the pore widths being larger than that necessary for effective capillary pressure.

Heaving is due to freezing of a relative large body of water in the fill. This body is formed by a small amount of water drawing to it many minute parts of water, thus growing in size. This drawing action is due to capillarity of the soil, and will occur, therefor, only in soils that will swell. Since freezing weather is necessary for this action, no heaving will occur in Santa Clara Valley.

Settlement in an earth fill dam is not important if it occurs in moderate amounts. It is evident that it is simply a further compacting of the soil. If the soil is such that voidal space is low and internal friction and cohesion high, very little settlement will take place.

The Bureau of Public Roads (9) finds that the most stable soil is one that is well graded; that is, contains about 50% gravel, and in the soil mortar, (that fraction passing the No. 10 sieve), 70% to 80% sand, 10% to 20% silt, 5% to 10% clay. These proportions may be widely varied yet the soil will remain stable, providing the grading and the structure remain such that the fines will bind the coarse material together, fill the voids and exhibit little or no capillarity.

The forces which cause water to pass through a soil are, the head of water acting on the soil and capillary pressure. Darcy's formula expresses the relation for flow due to the head. It is:-

$$Q = k F s$$

Where:

Q = volume of flow per year in cu. ft.
F = cross sectional area in sq. ft.
s = hydraulic gradient in ft. per ft.
k = coefficient of permeability in cu. ft. per yr. per sq. ft. under a hydraulic gradient of unity

Thus the permeability coefficient is the criterion of imperviousness of soil. It is dependent upon the area of the channels of flow and the resistance which these channels offer to flow. The channels of flow are the voids existing between the particles. These voids may be thought of as small broken tubes through which the water flows. The smaller these voids in cross-sectional area, the greater the resistance to flow, due both to the great increase in wall resistance, and to the tortuousness of the path. The smaller the number of voids, the fewer the number of paths of flow.

To obtain a small voidal space it is necessary, as was seen above, to have a material in which the grading is such that there are sufficient fines to fill the voids between the larger particles, and to compact the material by an external force. The voids in compacted clay are very small in cross-sectional area, but the percentage of voidal space is relatively high (45% to 55%). Thus in clay there are many channels of flow but each channel is very small and offers high resistance to flow. A well graded material when compacted will have as low as 25% to 35% voids, but the cross-sectional area of each voidal tube will be greater than that of clay. Due to the fact that clay compacts more readily and under wider moisture conditions than well graded material, it is usually thought that clay has a lower permeability coefficient than properly compacted graded material. Tests by Proctor (3) show that this is in error. This is due to the larger voidal space overcoming the advantage of a smaller cross-sectional area of the flow tubes, and to the capillary pressure existing in the small flow tubes of clay. For a discussion of Soil Mechanics, see the treatise by Dr. Terzahgi (10).

Since capillary pressure varies inversely as the diameter of the tubes, it becomes very high for clay, and forces considerable water through the soil. In well graded material, due to the presence of sand, the voidal tubes do not become excessively small; therefore capillary pressure is usually negligible. Thus, in order to have a low permeability coefficient, the soil must have a small percentage of voids, the resistance that these voids offer to flow must be high, and capillary pressure kept at a minimum. This is accomplished by selecting material that is graded so that when compacted the fines will fill the voids between the larger particles to the greatest possible extent without cuasing capillarity.

Since internal friction, cohesion, degree of compaction and swelling vary with change in moisture content, the stability of the soil will vary with moisture content. Therefore, a soil which is stable when placed will remain stable if it is impervious though subjected to outside moisture. It is further noted, that a soil consisting of gravel, sand, silt and clay, is stable when the voidal space is low, the same condition necessary for a low permeability coefficient. Soils consisting mainly of clay, silt, peat, diatoms, colloids and mica are impervious to gravity flow of water, but their high capillarity may cause them to become unstable, especially due to swelling.

Therefore, constructing a fill or using abutments of foundations that consist of gravel with a good soil mortar, results in an impervious and stable dam, provided the material is compacted to a dense mass.

GENERAL PURPOSE OF THE LABORATORY

Before the dam can be designed, the general characteristics of the soil in the borrow pit area must be determined. This preliminary testing was done by the Pacific Hydrologic Laboratories of San Francisco. They performed specific gravity, compaction, workability, grading and permeability tests. Since soil characteristics vary, even in a small locality, it is necessary that a close control be exercised over soil selection and placement while the fill is being constructed. To accomplish this, the District established its own laboratory to function during construction, using simplified and less elaborate testing methods as given below.

The functions of the District laboratory are divided into two divisions; the first dealing with soil tests; the second dealing with all tests other than soil.

Under the soil division, the laboratory must determine:

1. The grading of the soil in order to judge its suitability for the impervious or pervious section of the fill.
2. The moisture content which the soil must have in order to be compacted to its maximum unit weight with the existing rolling equipment.
3. The condition of the fill as placed, in regard to its compaction and moisture.

The grading of the soil is not a complete criterion of its suitability for the impervious or pervious sections of the fill. However, as was seen above, it is the most important, as it determines the density to which the soil may be compacted, and the size of the flow tubes. Lane (1) of the Bureau of Reclamation has studied the grading of materials in existing earth fill dams. By performing numerous percolation tests on samples of the various materials, and noting the behavior of these materials in the dams, he has arrived at an approximate limit of materials suitable for impervious sections of rolled earth fill dams, see Fig. 3. It was found that the material whose grading is finer than the limit line is of such a nature that if compacted to a dense mass, leakage through it is small enough to prevent erosion. If the material contains a great amount of clay, capillarity and unstability will be very important; therefore, the limit set down by Lane does not hold for very fine material. Lee of the Pacific Hydrologic Laboratory, proposes a fine limit which would be a straight line from 100%, passing the No. 28 sieve to 30% finer than 0.005 m.m., on Fig. 3. Since the materials proposed for use in the District's dams contained, in all cases, pebbles and gravel, the use of Lane's criterion is satisfactory. Therefore, by determining the grading of the soil and comparing it to the limit of grading, an indication of its suitability for the impervious or pervious section of the fill is obtained. In other words, determination of grading takes the place of the more elaborate percolation tests.

The moisture content which the soil must have so that it may be compacted is determined by the laboratory compaction test.

The condition of the fill is determined by the field compaction test.

Under the second division of the functions of the laboratory, comes cement, concrete aggregate, concrete, structural steel, and reinforcing steel tests. Since the laboratory was not equipped with a tension or compression machine, these tests were sent out. The University of Santa Clara performed all steel, cement and 7-day concrete tests up to the limit of capacity of the testing machine, while Robt. W. Hunt Co;, and Abbot A. Hanks, Inc. performed all 28-day concrete tests. All these tests and the test reports were handled through the laboratory. Concrete aggregate was screened in the laboratory and all concrete mixes were designed by the testing engineer.

DESCRIPTION OF THE LABORATORIES

The first dam to be built was Vasona. However, owing to the small size of the dam, which was nothing more than a levee on each side of a concrete spillway structure, and the uniformity of material, no soil testing was done. Concrete mix design and testing was handled by the Testing Engineer, working out of the Office of the Chief Engineer in San Francisco.

In the middle of January, 1935 work was started on the Calero dam. A laboratory, in charge of the Testing Engineer, was established in an abandoned farm house near the Calero dam. The testing engineer then used this laboratory as headquarters. About the first of March exploration work was started at Coyote. A laboratory was established at Coyote under the direction of the Assistant Geologist, Roger Smitten. The reports of tests of all nature from Coyote were handled by the Coyote laboratory.

Work at the Guadalupe and Stevens Creek sites started in June. At that time an assistant was added to the laboratory staff. In July, when work on roll fill began at both Guadalupe and Stevens Creek, the laboratory was moved from the Calero dam site to the Grant Building in San Jose, so that it would be more accessibly located. As work progressed at Calero and Guadalupe, it was decided that moisture tests on fill material should be performed at the sites to insure closer control. Near the end of July facilities were installed at these sites to make moisture tests. A man was placed in charge of these tests at each dam. When roll fill started at Almaden in August, moisture tests for that job were run at Calero until the completion of the Calero fill in September, when they were then run at Almaden. The moisture tester at Guadalupe divided his time between Guadalupe and Almaden. With the completion of the Guadalupe fill in October all Almaden tests were performed at the San Jose laboratory. All of the Stevens Creek tests were performed in San Jose, also.

LABORATORY TESTS AND PROCEDURE

Compaction Test:- Laboratory

To determine the moisture content which the soil must contain so that it may be compacted to its greatest density, that is, its "critical moisture", the following test is carried out: A representative sample of about 60 pounds is taken from the borrow pit. The material is put through the following square screens: 1½ inch, 1 inch and 3/4 inch. The amount remaining on each after all lumps are broken is recorded in pounds and ounces to the nearest one-half ounce on the Fairbanks Scale, No. 528. The material passing the 3/4 inch screen is quartered so that a sample weighing about a pound and a half is obtained. The sample is set aside for further screening as will be explained later. The rest of the material is then put through the No. 4 Tyler Standard Screen. Care must be taken that all lumps are broken. The material passing the No. 4 is then fully dried in the Ivanhoe Oven, the baking pans being used to hold the material. After drying it is allowed to cool so that it recovers its hydroscopic moisture. When coll, five pounds of the material is weighted out, a known amount of water is added, and this is mixed so that all the material has the same moisture. This moist material is then placed in the compaction cup. The compaction cup is made in three parts; a base plate, a cylinder and a collar. The cylinder had a top internal diameter of 4-1/16 inches, a bottom diameter of 4-3/16 inches, a depth of 4 inches and a volume of 0.0309 cu. ft., and is made of 1/4 inch steel. The cylinder fits into the baseplate 1/16 of an inch in a running fit and is held there by three flanges. The base plate is 8 inches square and 3/8 inches thick, machined to receive the cup. The collar, which is to protect the top of the cup, has the same diameter as the top of the cylinder and is 1-3/8 inches in depth. It is held on the cylinder by four flanges which hook on to pins in the cup. The tamping rod is a piece of 1-1/4 inch steel shafting, 16 inches long and weighing 5½ pounds. The moist material is placed in the compaction cup in three equal layers, each layer being tamped 25 times by the tamping rod with a firm 12 inch stroke. This tamping from test has been found to be equivalent to the field compaction equipment on the dam. After the third tamping the collar is removed and the compacted soil cut off with a trowel level with the top of the cylinder. The cylinder and base plate with the soil is then weighed. The dry weight of the soil per cubic foot as compacted is obtained by the formula:

$$W_d = \frac{W - W_c}{V(M + 1)}$$

Where:

W_d = dry weight of soil in pounds per cubic foot

W = weight of soil, cylinder and base plate in pounds

W_c = weight of cylinder and base plate in pounds

V = volume of cup in cubic feet

M = % moisture = weight of water added, divided by dry weight of soil used.

The procedure is carried out for moistures varying between 4% and 20%, or as many as necessary. The resulting dry weights, as a function of moisture in per cent of dry weight, are plotted on a graph as in Fig. 4. All soils have the same general characteristics as is shown by the curve on Fig. 4. The maximum compacted weight varies with each material, and is higher for those materials which compact at a low moisture. The maximum point on the compaction curve gives the moisture at which the material must be compacted to obtain the greatest density. The tamping should be such as to simulate actual field compaction by the rollers. The above procedure is essentially the same as outlined by Proctor (2). This test is run when a new borrow pit is opened and requires about 8 hours to perform.

Plasticity:

Proctor (2), in his work for the Bureau of Water Works and Supply of the City of Los Angeles, developed a field method of testing for compaction. It is based on the theory that soil compacted to different dry weights due to variation in moisture content will offer different resistances to penetration. As the moisture content varies, the internal friction ahd cohesion will vary; thus changing the plasticity of the soil. This plasticity is measured by a device known as a plasticity needle. It is built like a bicycle pump with a needle of known cross-sectional area screwed on the bottom. Pressing down on the handle stresses a calibrated spring, forcing the needle into the soil. The pressure required to force the needle so that it gives a jump due to the pressure built up, or to force it about 2 inches into the soil, divided by the cross-sectional area of the needle, gives the penetration resistance in pounds per sq. in. The pressure exerted on the spring is indicated by a sliding indicator on the handle. By performing this test on the compaction test samples, a curve of penetration resistance against per cent of moisture may be plotted as in Fig. 4. By using this result and the results of the compaction test, the penetration resistance for maximum compaction is determined as they both occur at the same moisture content.

Field Compaction Test:

To determine if the material is rolled sufficiently, it is necessary to determine the dry weight of the compacted material in the fill. This is done by the following method:

After thespread has been watered and rolled, the loose top soil is removed from a two ft. square area. A 12 inch ring, 1½ inch deep, made of ½ inch steel, with a beveled cutting edge, is driven into the fill so that the material enclosed by the ring is level with the top. The material inclosed by the ring is removed by the 6 inch auger. The object is to go deep with a 6 inch hole rather than to remove all of the top soil in the ring so that about 30 pounds of fill is removed. The removed material is carefully weighted to the nearest quarter pound on the Fairbanks Scale, No. 12006, and it is then quartered for a moisture and screen test. Into the hole dry sand is poured, not tamped, so that the hole is filled level to the ring. The principle object of the ring is to enable the hole to be filled with sand to the exact level of the removed earth. The weight per cu. ft..of sand is known by test; that is, by weighing one cu. ft. of the sand as poured into the cubic foot box. No. 2 Olympic sand was found to be very satisfactory. The weight of sand used is determined by the difference in weight between the original sand in the container and that remaining after pouring. The ratio of sand poured into the hole to its weight per cu. ft. gives the volume of the hole in cu. ft. The dry weight of the soil as compacted in the fill is determined by the formula:-

$$W_d = \frac{W\ D}{(1 + M)\ W_s}$$

Where:

W_d = dry weight of soil as compacted per cu. ft.

W = weight of soil removed from hole

D = weight of sand per cu. ft., dry loose

M = Moisture in soil removed in percent of dry weight

W_s = weight of sand poured into hole

If the material is properly rolled, the weight in the fill as calculated by the above formula should agree with the weight as determined by the compaction cup for the same moisture and material. This field test then gives a very close check on the efficiency and thoroughness of the rolling operation, and it is one of the most important of the tests performed.

The frequency of this test depends upon conditions in the field. It was run at least once each shift as it enables the inspectors to determine with certainty the degree of compaction of the fill. The field compaction test was run at Calero about every 2,100 cu. yds. of fill, at Guadalupe every 3,000 cu. yds., at Stevens Creek every 5,000 cu. yds., at Almaden every 2,200 cu. yds., and at Coyote every eight hours. The inspector, or moisture tester, performed this test.

Moisture:

To obtain the maximum compaction it is essential as was pointed out above, that the moisture content of the soil be closely regulated. The following method was used to determine the moisture:

The material taken out of the hole in the field compaction test is quartered so that about a 9 lb. sample is obtained, the size depending upon the rocks present. This sample is then sent to the field laboratory or the San Jose laboratory, depending on the job, in a one gallon paraffined fibre ice cream container. Paraffined containers were used to prevent loss of moisture. In the laboratory, the wet material is screened through the 3/4 in. screen. That soil passing the 3/4 in. screen is quartered down to about 1½ lbs. and is called the moisture sample. The moisture sample is carefully weighed on the Harvard balance to the nearest 0.10 gram and then dried out in the Ivanhoe oven for about twenty minutes. It is then allowed to cool and regain its hydroscopic moisture, after which it is again weighed. The loss in weight represents the moisture, or the percentage of moisture, which is calculated:

$$M = \frac{W_w - W_d}{W_d} \times 100$$

where:

M = moisture in per cent of dry weight

W_w = weight of moisture sample wet

W_d = weight of moisture sample dry

The moisture calculated by this method is the recorded moisture, and is called the moisture content of the fill. The object in screening the material through the 3/4 in. sieve to obtain the moisture sample, is to obtain a percentage of moisture that is based on as near similar conditions as that used in the laboratory compaction test where the moisture is calculated on material passing the No. 4 sieve. The moisture necessary to lubricate fine material is much greater than coarse material due to its greater surface area per unit weight. To base the moisture content on exactly the same conditions as used in the laboratory compaction test, the sample would have to be screened wet through the No. 4 sieve, an operation that takes considerable time due to the stickiness of the wet material. To speed up the operation, the material passing the 3/4 in. screen is used to determine the moisture content for field samples. Further, in calculating the dry weight from the field compaction test, it is necessary to know the moisture content of the total sample, including rocks above the 3/4 in. screen. If the moisture sample was based on material passing the No. 4 sieve, this percentage moisture would be greatly in error for use in field compaction tests due to the rocks which have a very much lower surface area per unit weight. When the moisture is based on material passing the 3/4 inch screen, the material above the 3/4 inch screen may be considered as dry, with very little error, especially for impervious material. Therefore, the percentage of moisture used in calculating the field compaction is obtained by multiplying the percentage of moisture of the moisture sample, by the per cent based on dry weight of material passing the 3/4 inch screen as obtained by the grading test.

Grading:

To determine the grading of the material so that it may be compared with the limit as set down by Lane (1) it is necessary to separate the soil particles into their respective sizes. This is done by screening and sedimentation.

The same procedure is followed for material from the fill or borrow pit. If taken from the borrow pit, a sample large enough to be representative is sent to the laboratory, usually about nine pounds, except when a compaction test is to be run, when about sixty pounds is sent as was explained under "Laboratory Compaction Tests". The material from the fill is taken when the field compaction test is made. The nine pound sample is used for both moisture and screening.

The following method of screening is used:

The sample as received from the field is screened through the 1½ inch, 1 inch and 3/4 inch screens. The amount retained on each is weighed to the nearest one-half ounce on the Fairbanks Scale No. 528 and recorded. The material passing the 3/4 inch screen is quartered for the moisture sample. The moisture test sample, after drying, cooling and weighing is used for the remaining screening and sedimentation. The dry moisture sample is screened through the 3/8 inch screen, and the following Tyler Standard Screens, Nos. 4, 8, 14, 28,48, 100 and 200 mesh. The amount remaining on each screen is carefully weighed on the Harvard Balance to the nearest 0.10 gram and recorded. It is absolutely necessary in screening soil that all lumps be broken up so that the grading represents particle size. This is done by hand for the material on and above the 3/4 inch screen. For that material below 3/4 inch the following method is used:

After drying, the soil is placed in a baking pan and the larger lumps broken by tamping with a 2" x 4" post. This material is then screened through the rest of the sieves. Many small lumps , however, remain on each screen, which must be broken. The small lumps on the 3/8 inch, No. 4 and No. 8 screens are broken by placing the material remaining on the screen in a baking pan and crushing each lump by means of a garden trowel. The lumps smaller than the No. 8 are broken up by placing the material held on each screen in another baking pan and grinding the material by means of the garden trowel from which the handle has been broken. The trowel is used like an assayist's pestle. In grinding and crushing, great care must be exercised so that the

Particles themselves are not broken and abrasive action is kept to a minimum. The shaking is done by hand, and care must be taken to insure that all the particles smaller than the screen opening are shaken through.

To determine particle sizes smaller than the 200 mesh sieve, the sedimentation principle is used. The particle size and quantity is determined by the Bouyoucos hydrometer. Fifty grams of the material passing the 200 mesh sieve is placed in the dispersing cup. This cup is the same as is used in soda fountains for milk shakes, except baffles are placed on the inside. To the fifty grams of soil, distilled water is added, filling the cup within an inch of the top. If the material will floculate, 5 c.c. of sodium oxalate and 5 c.c. of sodium hydroxide are added as a defloculating agent. The mixture is then stirred for 10 minutes in the Hamilton-Beach stirring machine. The dispersed soil is then poured into the special graduated glass jar and distilled water is added to bring the level of the mixture to the proper height. The dispersed soil is then shaken by hand in the cylinder for about a minute. As soon as the jar is set down, the hydrometer is placed into the mixture and its reading taken after 40 seconds. This reading of the hydrometer gives the grams of soil in suspension that have a particle diameter of less than 0.05 m.m. At the end of 24 minutes the hydrometer is read again for soil with a particle diameter less than 0.01 m.m, and again at the end of 60 minutes for the soil below 0.005 m.m. diameter. The hydrometer is removed after the 40 second and 24 minute reading. As the hydrometer is calibrated at 67° F., a correction of 0.10 is added to the hydrometer reading for every degree above 67° F., and subtracted for every degree below 67° F.

The results of the screening and hydrometer tests are expressed as cumulatige percentage of dry weight retained for each size. The dry weight of the sample is determined by the moisture test, considering the 3/4 inch and larger rock as dry when received, as is done for the field compaction test. The total dry weight of the sample is then:

$$W_d = W_c + \frac{W_p}{1 + M}$$

Where:

W_d = dry weight of sample from field

W_c = cumulative weight of material retained by 3/4 inch screen

W_p = weight of wet material passing the 3/4 inch screen

M = percent of moisture as determined by the moisture test

The cumulative percentage for each size is plotted on forms as shown in Figure 3. In calculating these percentages it must be remembered that the percentage is based on the dry weight of the sample as taken from the job, and consequently, the calculations must take account of the various quarterings.

Screen tests were performed with the following frequency:

Almaden - About every 2,800 cu. yds. of fill
Calero " " 3,500 " " " "
Guadalupe " " 4,500 " " " "
Stevens " " 6,200 " " " "
Coyote - Every eight hours

Twice as many samples were taken from the impervious sections as the pervious, except at Stevens Creek where it was about equally divided. The time required to perform a single moisture, grading and sedimentation test is about four hours.

Concrete Design and Testing:

Concrete was designed using a combination of water-cement ratio and maximum density. The concrete aggregate was screened, and the grading of the large rock, small rock and sand obtained. Then by trial and error the proportions of each that should be combined to give maximum density was determined. The results were plotted on forms as shown in Figures 5, 6 and 7. The real mix was determined by using the curves of the Portland Cement Association, giving a relation between fineness modulus and real mix for various slumps. Due to the wide variation is moisture content of the aggregate, no attempt was made to determine the amount of water that should be added, the consistency of the mix being controlled by the Concrete Inspector with the aid of a slump cone.

So that the field mix could be obtained from the real mix, bulking tests were run on the aggregate in the usual manner, a sheet metal cubic foot container being used for this purpose.

Screen tests were run on the aggregates whenever, in the judgment of the Inspector or Testing Engineer, conditions warranted. By watching the aggregate and results of the concrete compression tests, the mix was adjusted from time to time to give a mix which would be economical to the contractor, yet meet all of the specifications.

Concrete test samples were taken about every 100 cu. yds. of pour during a long pour. On shorter pours the size and importance of the pour determined if a sample was to be taken. So that the compression tests results would be had as early as possible, all samples were tested at 7 days, and for every third test a pair was taken, one to be tested at 7 days, the other at 28 days; thus checking the ratios existing between 7 day test strenghts. The A.S.T.M. standards were followed in all concrete tests. The concrete tests were reported on forms shown in Fig. 11.

The concrete for testing was moulded in a Fibreboard Products Container, 5-7/8 inches in diameter and 12 inches long. The container was paraffined on the inside only. After setting for one day in the field, it was sent to the concrete testing laboratory. Those sent to Santa Clara University were delivered personally, while those for San Francisco were sent, packed in damp sawdust, by express. So that the bottom of the cylinder would be true, a machined steel disc, 5-7/8 inches in diameter were used:

Other Tests:

The steel and cement tests were subjected to the ordinary A.S.T.M. standard tests, samples being taken at the discretion of the Inspector or Testing Engineer.

Reporting of Tests:

The results of laboratory compaction tests, (Fig. 4) were sent to the Chief Engineer, the Supervising Field Engineer, the Resident Engineer, and one was kept in the laboratory. The results of the screen and hydrometer tests were plotted on the graph, (Fig. 3), five tests being put on one sheet. Two sheets were made; one sent to the Chief Engineer and the other kept in the laboratory. The results of the field compaction, moisture, screen and hydrometer tests were tabulated as in Fig. 8, copies being sent to those receiving the laboratory test reports.

Concrete test results, (Fig. 11), were sent to the Chief Engineer, the Supervising Field Engineer, the Testing Laboratory, the Resident Engineer, the P.W.A. Inspector and the Distirct's laboratory. Cement and steel tests were sent to the same offices.

When a third of a dam was completed, a summary for that third was made. This consists of an averaging of the percent moistures, field compactions and grading curves of all tests for the impervious and pervious sections. This was done for the first, second and third thirds.

indicates that they are suitable for the impervious section, but comparing this grading with the grading for the impervious section, Fig. 9 shows that this material is less impervious than the material in the impervious section. Coyote dam has no pervious section.

Compaction:

With the exception of Stevens Creek, which has rather uniform material, the material in the dams is a conglomerate of various types of soil, having very different textures, formations and specific gravities. Due to this, and to the fact that laboratory compaction tests were based on material passing the No. 4 sieve, and did not include all types of soil, average results of the field compaction tests cannot be compared exactly to the laboratory tests. In general, however, the dry weight as obtained by the field compaction test was from zero to ten pounds below the dry weight as obtained by the laboratory compaction test, for the impervious section. The dry weight for the pervious sections indicates very good compaction. The per cent moisture content of both sections is very near to that indicated by the laboratory compaction test, if allowances are made for the material retained on the No. 4 sieve. The reason the pervious section has better compaction than the impervious, is mainly due to the ease with which the coarse material is compacted, and that the grading of the pervious material approaches that ideal curve for maximum density for the given effective size. For maximum density the grading curve should be concave upward.

To get a better picture of the condition of the fill, the following table is given. The specific gravities of the materials are obtained from the report of the Pacific Hydrologic Laboratory and are only an approximation for the average material of each dam.

IMPERVIOUS SECTION

DAM	SPECIFIC GRAVITY	PERCENT AIR VOIDS	PERCENT TOTAL VOIDS	SATURATION MOISTURE
Almaden	2.73	6.2%	30.2%	15.9%
Calero	2.75	7.8	34.1	18.7
Coyote	2.72	4.8	26.8	13.4
Guadalupe	2.73	12.0	33.2	18.2
Stevens Creek	2.73	7.7	27.5	14.0

PERVIOUS SECTION

DAM	SPECIFIC GRAVITY	PERCENT AIR VOIDS	PERCENT TOTAL VOIDS	SATURATION MOISTURE
Almaden	2.73	5.6%	27.6%	14.0%
Calero	2.75	12.5	28.9	14.8
Guadalupe	2.73	12.7	32.3	17.5
Stevens Creek	2.73	9.1	25.4	12.5

With the exception of the impervious section of Calero dam, a comparison of percent of total voids and grading shows that the finer the material, the greater is its total voidal space. This is a condition to be expected because fine particles, (clay), due to their very small weight and high cohesion, do not shift position after first contact between particles, and consequently remain a spongy mass. The results in the above table and in Fig. 9 tend to show that the impervious sections of Calero did not have sufficient rolling for maximum compaction. The average compaction as given by Figs. 9 and 10 will be increased due to the weight of the dam causing consolidation on the soil in the lower section. Proctor (2) found that the dry weight per cubic foot of a soil under 100 feet of fill may increase as much as 10 pounds.

Permeability:

No mathematical relation has been found between the physical properties of a conglomerate soil and its permeability coefficient. Terzaghi (10) gives a relation between permeability coefficient, effective size and voidal space for clean sand and for clay, but these do not hold when sand and clay are mixed. However, it is known that the permeability coefficient does depend upon voidal space and grading. This relation is complicated by the fact that the greater the percentage of fines, the greater the frictional resistance to flow in each flow tube, but the total voidal space also increases with percentage of fines. There is no doubt that there is an optimum condition where voidal space and frictional are combined in just the proper amount to give minimum flow.

The results of Proctor's tests indicate that compaction is more important for imperviousness than percentage of fines. The per cent of total voidal space in any of the dams is not excessive. Calero dam, with the lowest compaction and voidal space of 34.1% and 21% of material passing the No. 200 sieve, may be compared to a sample of Proctor's in which the material passed the No. 200 sieve, the voidal space being about 31% and 21.7%. Proctor found that the permeability coefficient of his sample was 0.0710 feet per year, an extremely small rate. While this comparison is very rough as the other physical properties of Proctor's sample are unknown, it does give an indication of the permeability of Calero dam. Allowing a safety factor of 10, the permeability of Calero dam is still within the allowable limits. The percolation tests run by the Pacific Hydrologic Laboratory indicate that the permeability coefficient is very small for all material, but an exact comparison cannot be made as the average grading and compaction of the field tests are dissimilar to the grading, and compaction of the samples used by the Hydrologic Laboratory. The permeability coefficient of the impervious section of the other dams is no doubt less than Calero.

For the pervious section, if compaction is considered, as it must be, besides the grading, there is evidence to show that the pervious sections are, in reality, as impervious as the impervious sections. As an example Proctor performed a permeability test on a sample in which 8% of the material passed the No. 22 sieve, and the voidal space was 26%. The permeability coefficient of this sample was 0.0156 feet per year. From the meager test data this indicates that the permeability coefficient of the pervious section of Almaden, Calero and Stevens Creek is very low - about that of the impervious sections. The pervious material of Guadalupe is so similar to the impervious, that it also has the same permeability coefficient. This is not a serious fault as many dams are built which have only an impervious section, and there is very little chance for excessive pressure to be built up in the dam due to seepage.

The most likely place for an earth dam to leak is between the foundation and abutments, and the fill. There is a natural cleavage plane between the fill and the foundations or abutments. Seepage water tends to follow this path around the fill. To prevent this, a cut-off trench is used. The effect of the cut-off trench in stopping seepage, provided it is filled with material suitable for the impervious section, is dependent upon the method of placing and the material in which it is placed. No laboratory tests are used to test the bond between the fill in the trench and the abutments. Great care is taken in filling the trench, and it is subject in all respects to the approval of the State Geologist.

Stability:

The grading of the soil is a good indication of the stability of the fill, but it does not tell all of the Story. Comparing the grading of the soil in the District's dams to the grading of soil in other successful dams (1) it is concluded that the grading is suitable for a stable structure. None of the dams have an excess of one size particles, this allowing interlinking and a high internal friction. The clay content does not appear excessive, so swelling will be negligible. Proctor ran percolation and consolidation tests on soils containing much more clay than present in the District's soil, and found no evidence of swelling. Due to the presence of sand and gravel, capillary pressure causing flow is also negligible.

The pervious sections are more stable than the impervious sections, as the soil mortar of pervious sections approaches the grading set down by the Bureau of Public Roads (7) for the most stable soil mortar.

An indication of the future stability of the dams is given by the saturation moisture in the table on Page 30. The saturation moisture is the moisture content the fill would have if all of the voids were filled with water. If no swelling takes place, this moisture is the maximum moisture content that the fill can have. In all probability, it will never reach that value due to consolidation and the difficulty of driving out the entrapped air. Calero dam, with its rather low compaction, may absorb 18.7% moisture, an increase of 5.7%. From the few tests performed by the plasticity penetration needle, it is difficult to determine if this moisture will cause loss of stability. It is believed however, that it will not. Therefore, it is concluded for the fill that:

1) The permeability coefficient of the impervious section is sufficiently low to prevent erosion or serious leakage.

2) The permeability coefficient of the pervious section is as low as that of the respective impervious sections.

3) The grading and compaction of the soil is such that the fill is stable and will remain so, the pervious section more so than the impervious.

Steel and Cement Tests:

Both of these materials were tested in the usual manner as given by the A.S.T.M. All of these test results met the specifications.

Concrete Tests:

The method of design used, that is determining the aggregate mix by the use of the maximum density curve and amount of cement by the fineness modulus, proved very satisfactory. The inspectors were able by means of the slump cone to control the water cement ratio. The contractors, in special cases, were allowed to vary within moderate limits the aggregate mix, so as to obtain the desired workability, but the water cement ratio was kept constant. Since the greatest yardage was poured on a slope, without forms except skreds, it was to the advantage of the contractor to pour a dry mix. The usual field mix used was, 1 part of cement, 2.8 parts of sand, 2.0 parts of small rock and 1.7 parts of large rock. With this mix and a consistancy giving a slump of 2 to 2½ inches, the average consistancy, the test strength at 28 days was between 3,000 and 3,500 lbs. per sq. inch, well above the strength of 2,500 lbs. per sq. inch required by the specifications.

The use of the seven day test results to predict the 28 day results was also very satisfactory, as it gave the results very much sooner. On these projects the 7 day strength was about 53% of the 28 day strength.

Since the concrete tests were handled in accordance with the A.S.T.M. standards no new technique was developed. The strength found by tests were usually greater than was to be expected from the design data.

Aggregates:

As the aggregates, for the most part, came from well equipped screening plants, very little change was found in the aggregate grading. All aggregate tested passed the specifications and was of good quality.

Number of Tests:

The number of tests made are shown by the table to be found on the following page.

NUMBER OF TESTS

Project	Concrete	Concrete Aggr.	Earth Fill	Reinf. Steel	Cement	Concrete Pipe	Steel Pipe	Total No. Samples
Almaden Dam	28	3	137					168
Almaden Bridge	1							1
Almaden-Calero Canal	6	7		1		1		15
Calero Dam	29	6	235	4	4			278
Calero Auxil. Dam	4	1	24					29
Stevens Creek Dam	37	11	96		1		2	147
Guadalupe Dam	29	8	180	1			1	219
Coyote Dam	7	15	237	1	2		1	263

Grand Total 1,120

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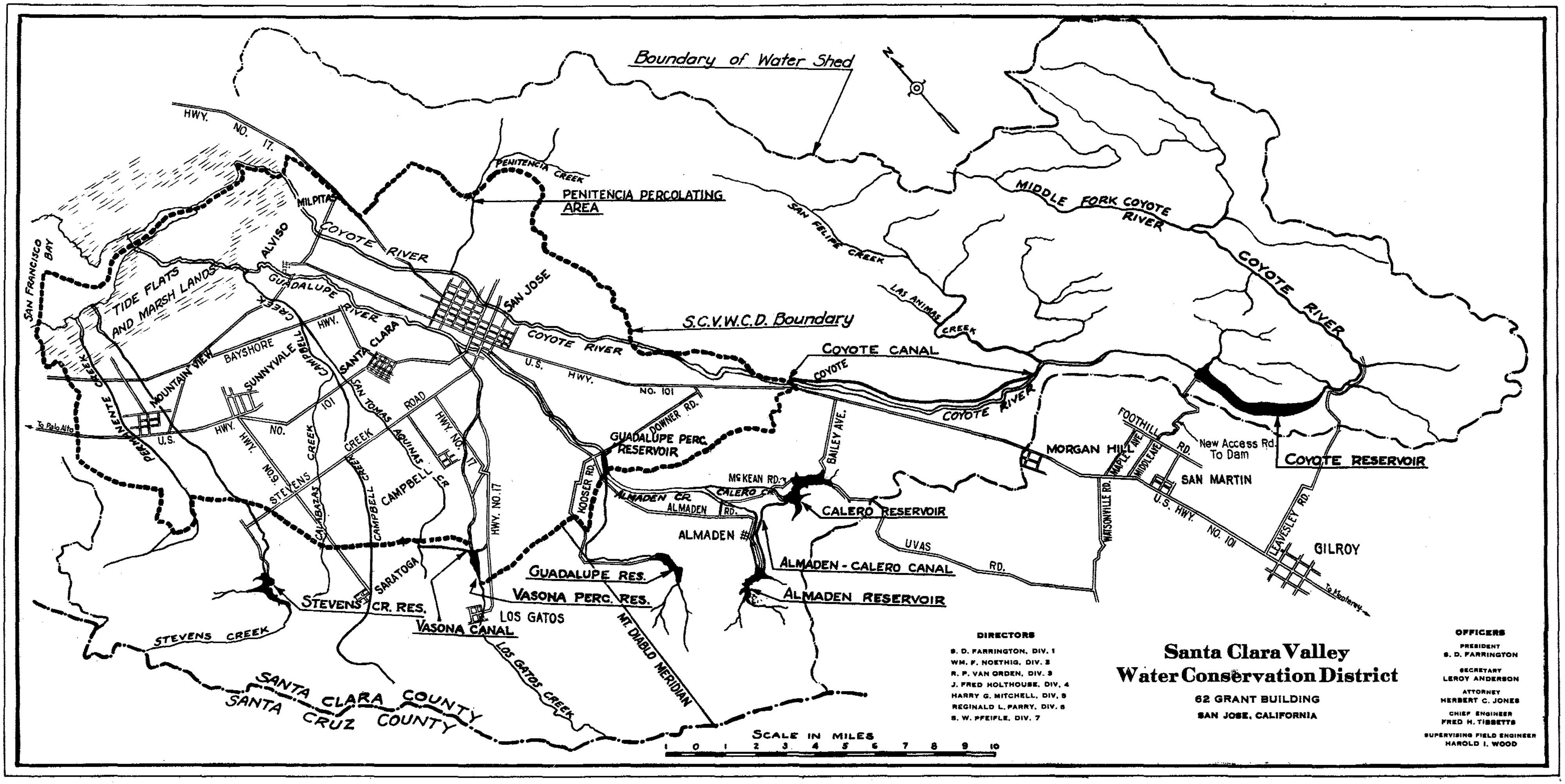
Santa Clara Valley Water Conservation District

130,000 Acres in Santa Clara Valley

Map Showing Detention Reservoirs and Main Highways Leading Thereto

Being Constructed in 1935



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R. P. VAN ORDEN, DIV. 3
J. FRED HOLTHOUSE, DIV. 4
HARRY G. MITCHELL, DIV. 5
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Santa Clara Valley Water Conservation District
62 GRANT BUILDING
SAN JOSE, CALIFORNIA

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LEROY ANDERSON

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FRED H. TIBBETTS

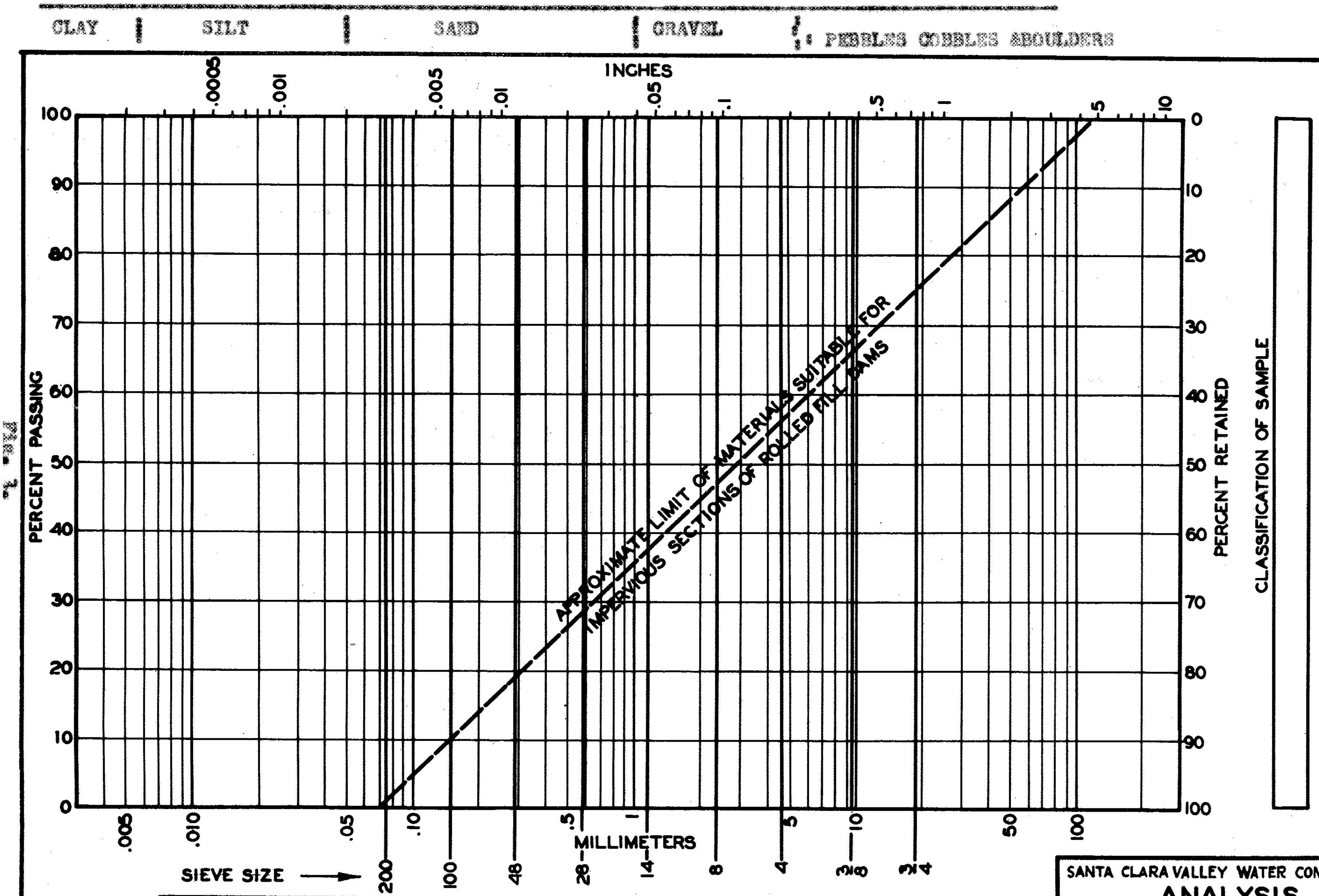
SUPERVISING FIELD ENGINEER
HAROLD I. WOOD

DATA ON SANTA CLARA VALLEY WATER CONSERVATION DISTRICT RESERVOIRS

	ALMADEN	CALERO	COYOTE	GUADALUPE	STEVENS CREEK	VASONA	SUMMARY TOTALS
Capacity of Reservoir- Acre Feet	2,000	9,500	30,000	3,500	4,000	750	49,750
Capacity of Spillway - Cu. Ft. per Sec.	7,000	5,000	33,000	6,000	10,000	12,600	
Area of Reservoir - (Max.) Acres	62	329	688	75	95	79	
Length of Reservoir - (Max.) Miles	1.1	2.2	4.8	1.1	1.1	0.8	
Type of Dam	Rolled Earth Fill	Rolled Earth Fill	Rolled Earth & Rock Fill	Rolled Earth Fill	Rolled Earth Fill	Rolled Earth Fill	
Height of Crest above Streambed - Feet	105	90	120	129	120	30	
Elevation Crest of Dam	615	490	798	627	545	305	
Dam Crest Length -Feet	460	840	990	650	1,000	1,000	
Dam Crest Width -Feet	20	20	100-160	20	20	20	
Dam Base Width -Feet	545	495	945	650	620	153	
Cubic Yards of Fill	250,000	550,000	1,060,000	520,000	530,000	70,000	2,980,000
Diam. of Outlet Pipe- Inches	36	36	50	36	50	42	
Length of Outlet Pipe- Feet	690	450	900	690	680	20	
Cubic Yds. of Concrete	3,000	4,000	3,000	4,000	5,000	2,000	21,000
Tons of Steel	130	170	120	130	200	150	900
Resident Engineer	J. W. Smith	R. P. Bryan	G. W. Hunt	T. D. Sawyer	F. W. Moore	J. W. Smith	
Contractor	F. O. Bohnett & D. McDonald	F. O. Bohnett & D. McDonald	Macco Construction Co.	A. Teichert & Son, Inc.	F. O. Bohnett & D. McDonald	Carl N. Swenson CO.	

Fig. 2

Fig. 2



SAMPLE NO.	DATE	LEGEND	COORDINATES		ELEVATION
			N	E	

SANTA CLARA VALLEY WATER CONSERVATION DISTRICT

ANALYSIS OF ROLLED FILL MATERIAL IN____________DAM

February 1935

Scale as shown

Fred. H. Tibbetts
Chief Engineer
San Francisco

Dr. C.B.P.	Ck. E.E.M.
Tr. E.S.R.	Ap.

File No.
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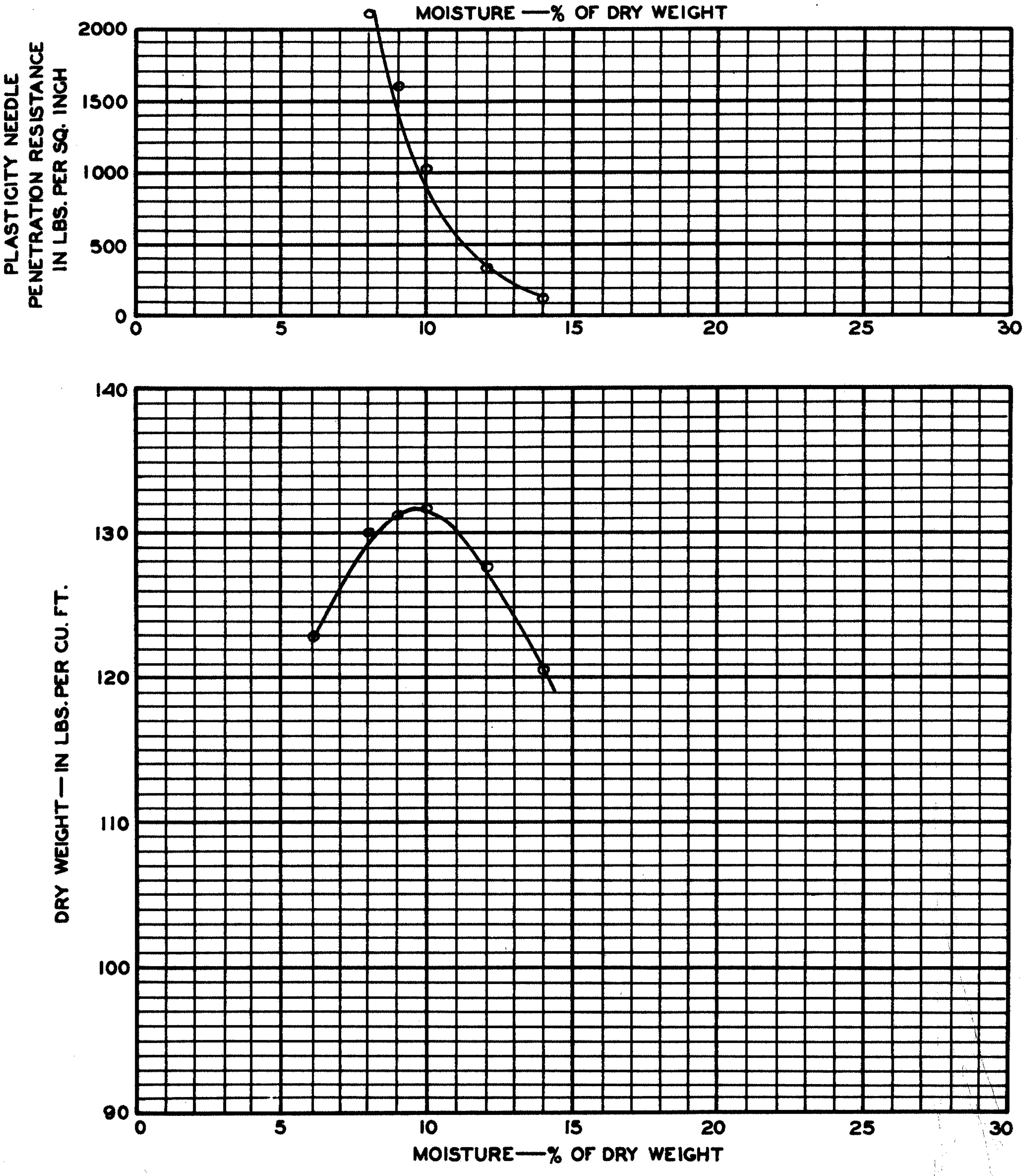


SAMPLE NO.	F-1147
DATE TAKEN	7-8-35.
DATE TESTED	7-10-35
TESTED BY	C. B. Palmer
% PASSING *200	15.0
DESCRIPTION	Gray Shale From Right Abutment

SANTA CLARA VALLEY WATER CONSERVATION DISTRICT

COMPACTION OF ROLLED FILL MATERIAL

________ DAM

Fred. H. Tibbetts
Chief Engineer
San Francisco.

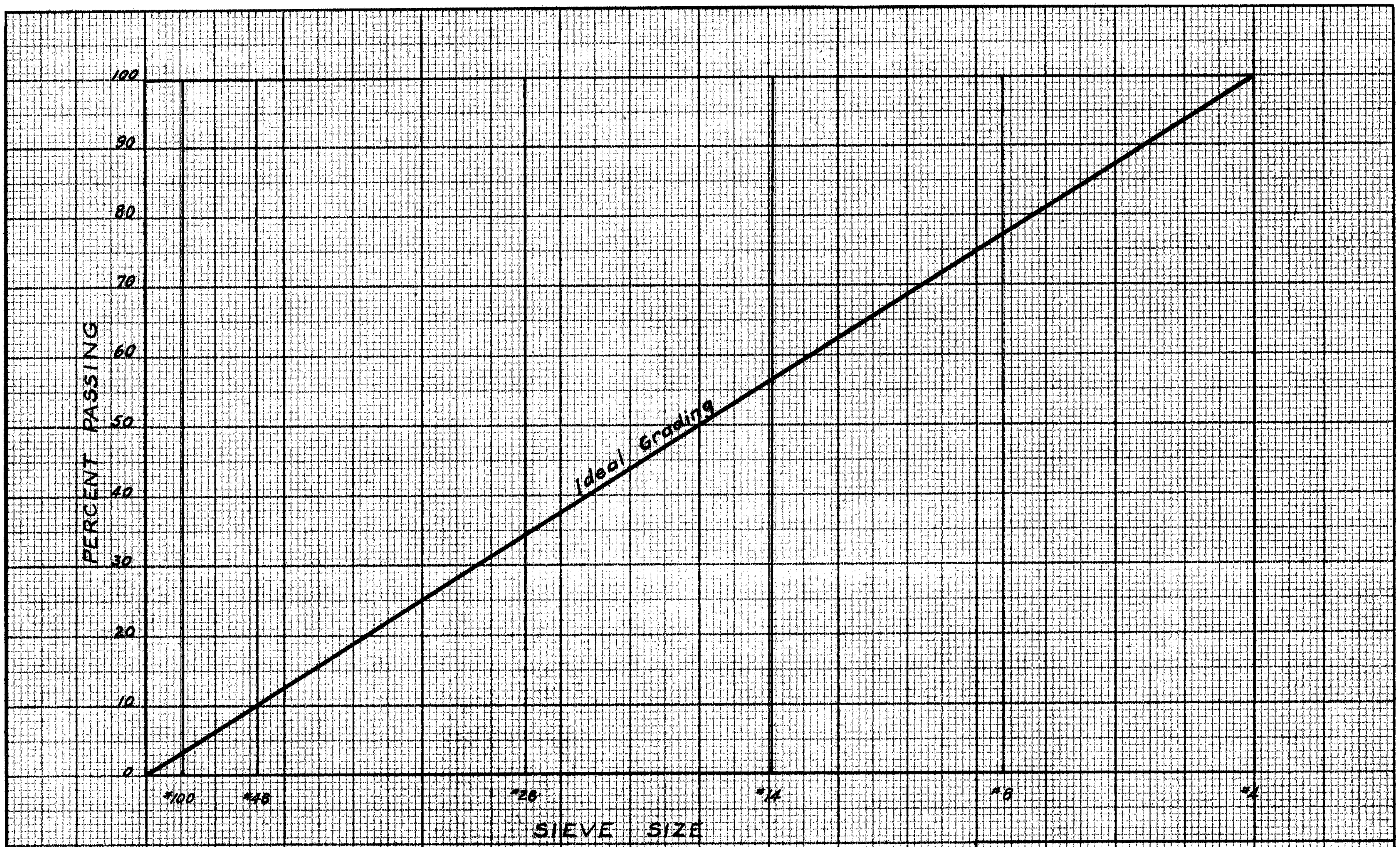
March 1935
Scale: as shown

Dr. C.B.P.	Ck. E.C.B.
Tr. E.S.R.	Ap.

Fig. 4

File No.
2022-47

Fig. 5



SANTA CLARA VALLEY WATER CONSERVATION DISTRICT

AGGREGATE GRADING CURVES
NO.4 MAXIMUM SIZE

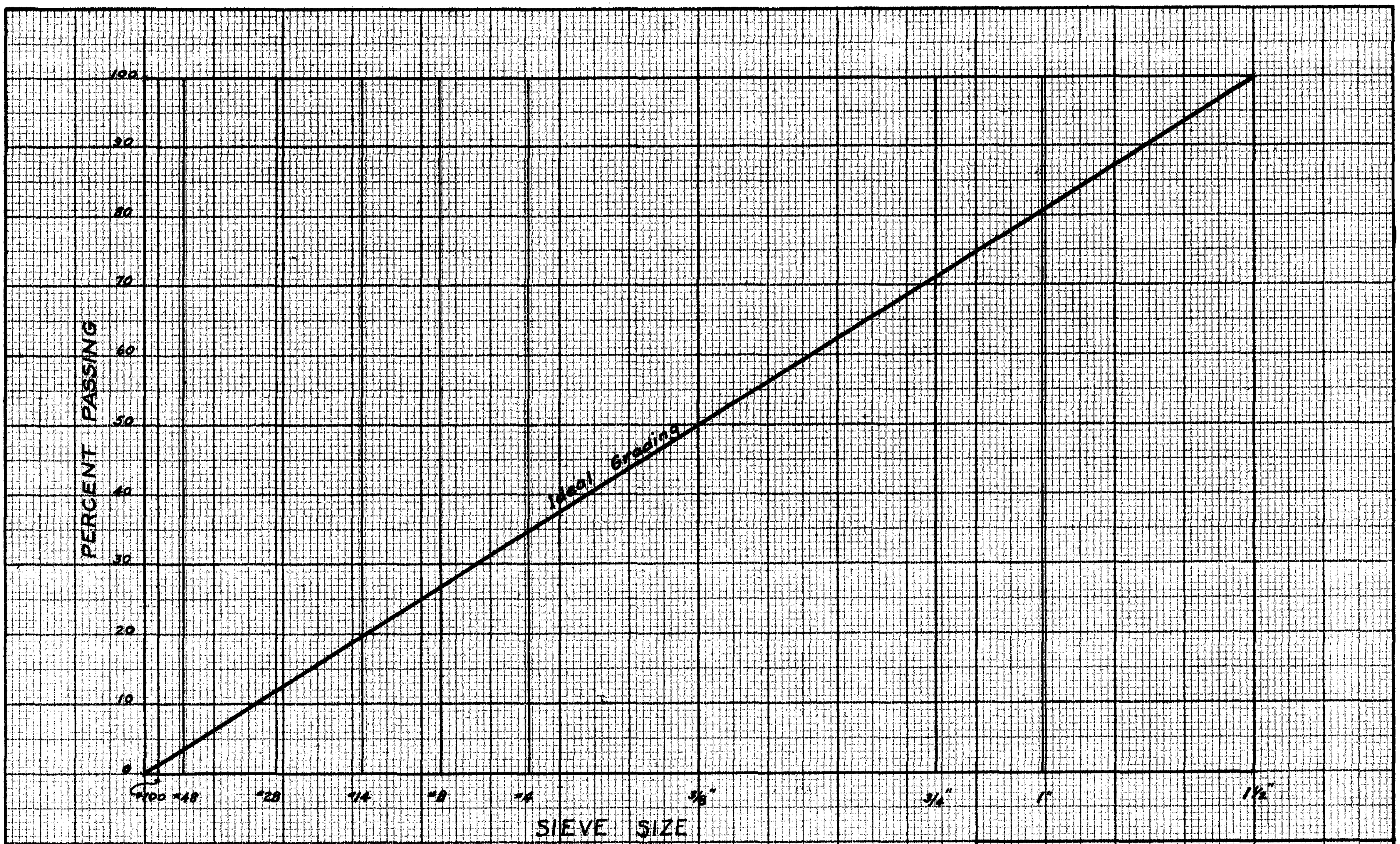
Fred H. Tibbetts
Chief Engineer
San Francisco

January 1935
Scale as shown

Dr. C.B.P. Ck.
Tr. Ap.

File No.

Fig. 7



SANTA CLARA VALLEY WATER CONSERVATION DISTRICT

AGGREGATE GRADING CURVES
1½" MAXIMUM SIZE

Fred H. Tibbetts
Chief Engineer
San Francisco

January 1935
Scale as shown

Dr. C.B.P.	Ck.
Tr.	Ap.

File No.

Alaska Commercial Building
San Francisco, Calif.

FRED. H. TIBBETTS
Civil Engineer

Fred. H. Tibbetts
Harold I. Wood
E. E. Blackie

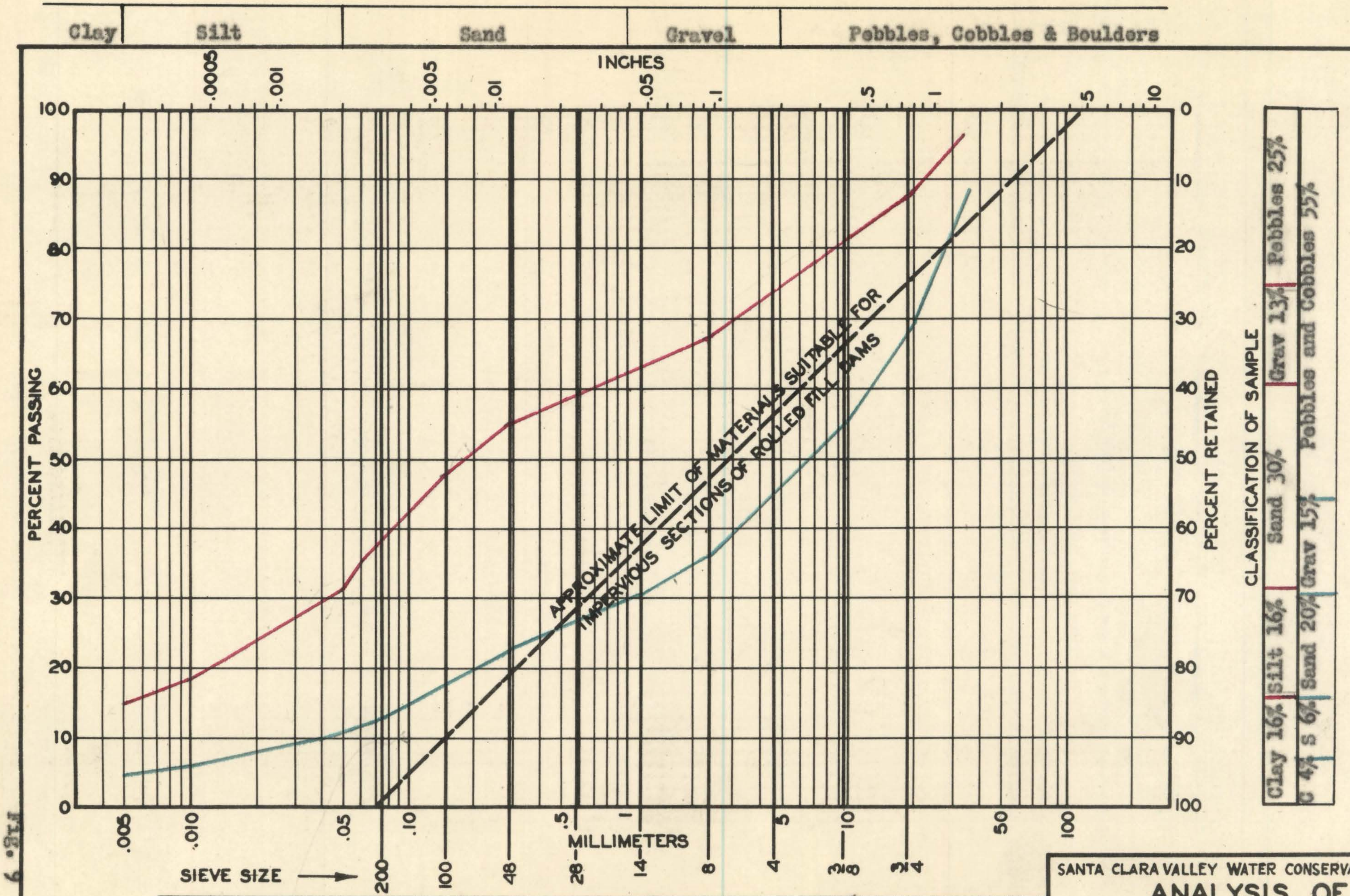
Project: S.C.V.W.C.D. Subject: ~~————~~ DAM
EARTH FILL TESTS

Computed by .... Date ....
Checked by .... Date ....
Sheet ...... of ...... Sheets

Number									
Date Taken									
Time									
Taken by									
Coordinates { N									
E									
Elevation									
% Moisture									
Wt. lbs./ Cu. ft. (dry)									
Sieve Analysis									
% Retained on	1½"								
	1 "								
	3/4"								
	3/8"								
	# 4								
	8								
	14								
	28								
	48								
	100								
	200								
	.05 m.m.								
	.01 m.m.								
	.005 m.m.								
Section									
Borrow Pit									
Remarks									

Fig. 8.

Fig. 9



SAMPLE NO.	DATE	LEGEND	COORDINATES		ELEVATION	% Moisture	Dry Wt. Lbs/cu.ft.
			N	E			
Average for all samples pervious					—	11.1	123.0
" " " " im pervious					—	12.6	118.7

SANTA CLARA VALLEY WATER CONSERVATION DISTRICT

ANALYSIS OF ROLLED FILL MATERIAL IN ALMADEN DAM

February 1935

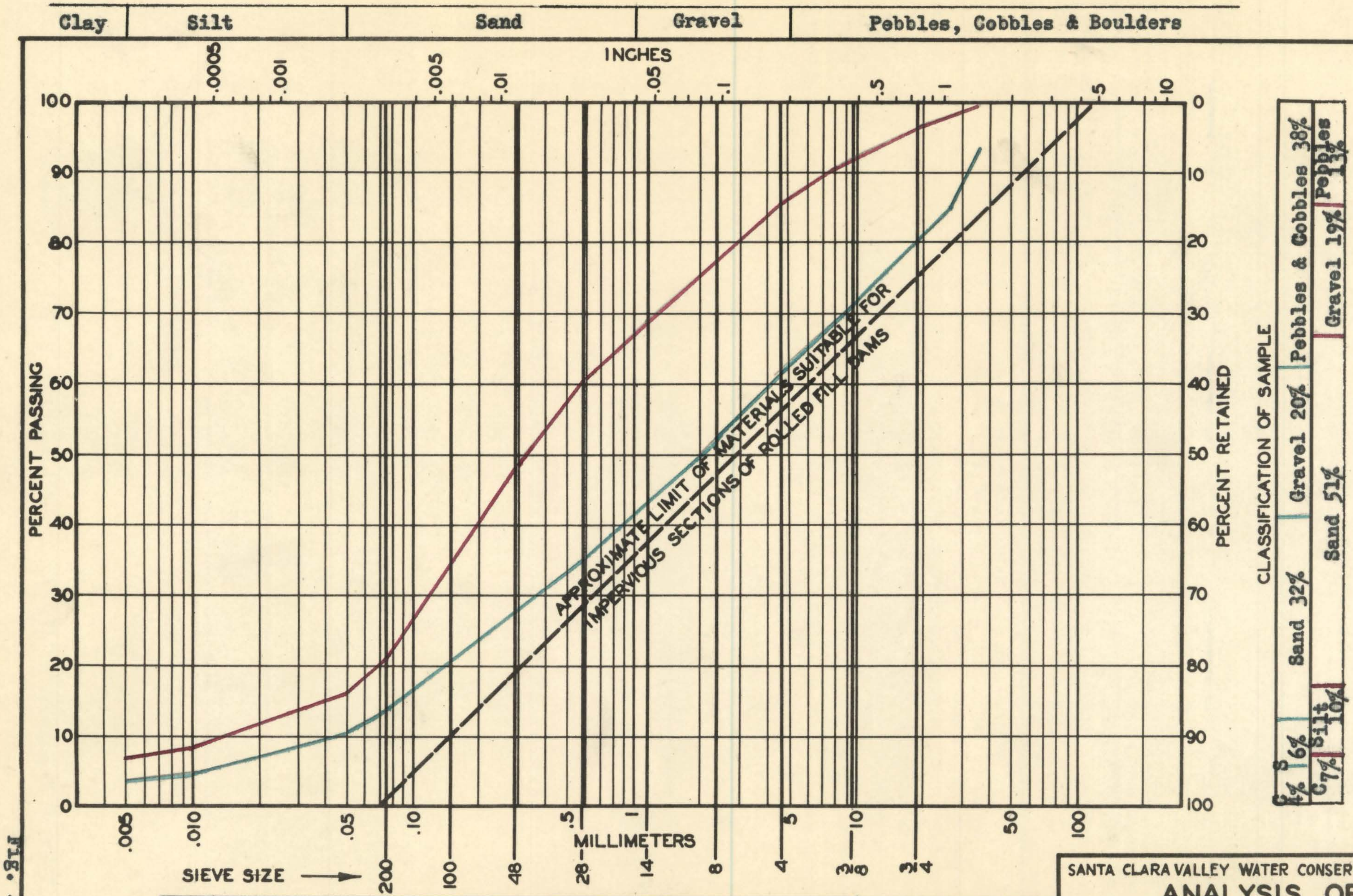
Scale as shown

Fred. H. Tibbetts
Chief Engineer
San Francisco

Dr. C.B.P.	Ck. E.L.C.
Tr. E.S.R.	Ap. F.H.T.

File No.
2022-46

FIG. 10



SAMPLE NO.	DATE	LEGEND	COORDINATES N	COORDINATES E	ELEVATION	% Moisture	Dry Wt. Lbs/Cu.Ft.
Average for all	samples pervious			—	---	8.4	122.0
Average for all	samples impervious			—	---	13.0	113.0

SANTA CLARA VALLEY WATER CONSERVATION DISTRICT

ANALYSIS OF ROLLED FILL MATERIAL IN CALERO DAMS DAM

February 1935

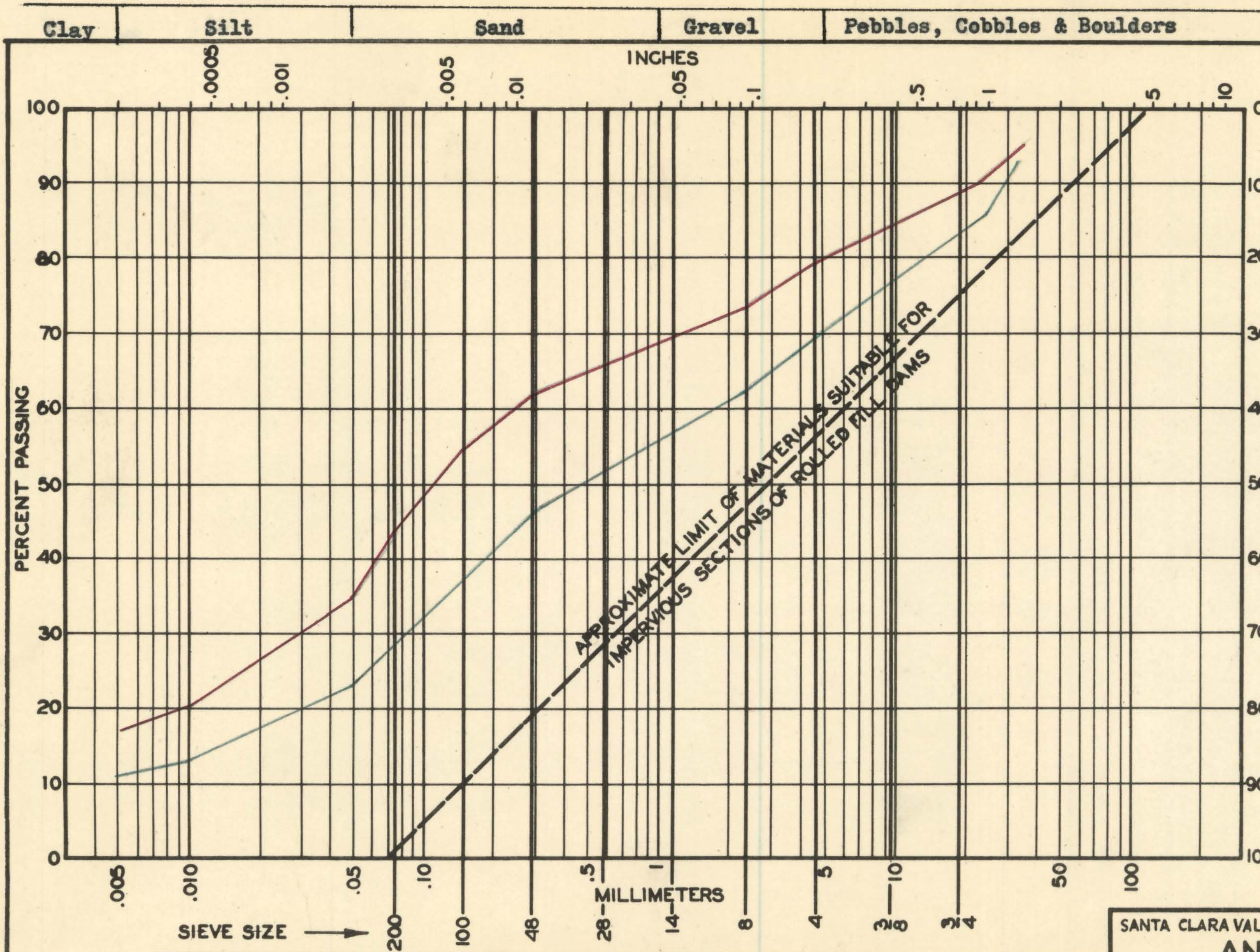
Scale as shown

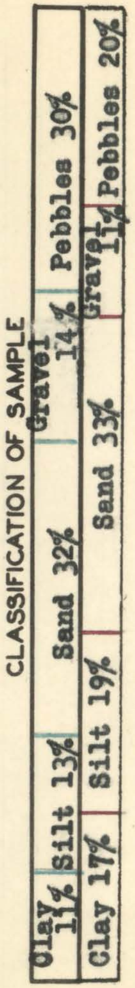
Dr. C.B.P. Ck. E.E.G.
Tr. E.S.R. Ap. F.H.T.

Fred. H. Tibbetts
Chief Engineer
San Francisco

File No.
2022-46

Fig. 11





SAMPLE NO.	DATE	LEGEND	COORDINATES N	COORDINATES E	ELEVATION	% Moisture	Dry Wt. Lbs/Cu.Ft.
Average for all Samples pervious					▬	10.6	115.1
" " " " impervious					▬	11.6	113.6

SANTA CLARA VALLEY WATER CONSERVATION DISTRICT

ANALYSIS OF ROLLED FILL MATERIAL IN GUADALUPE DAM

February 1935

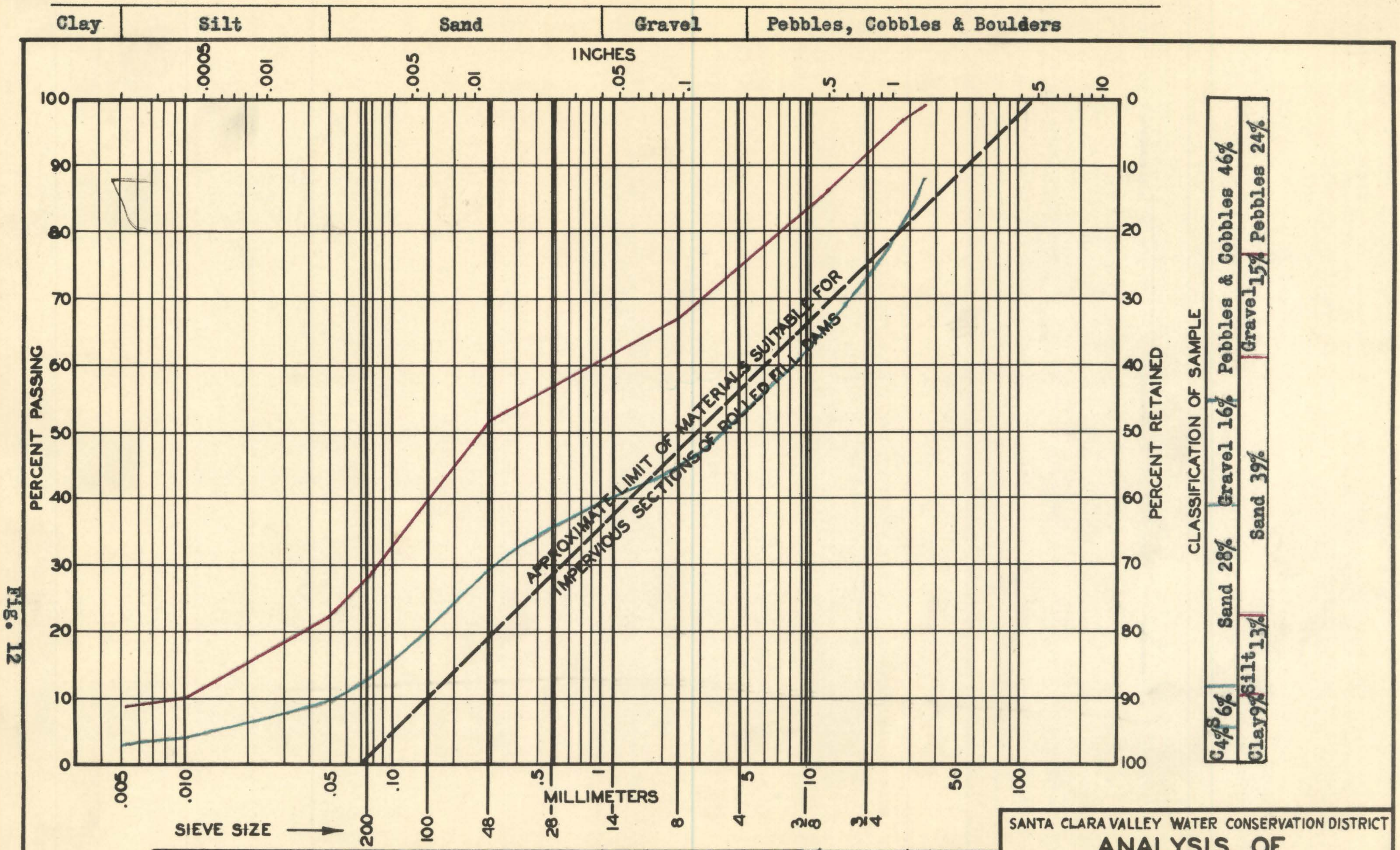
Scale as shown

Fred. H. Tibbetts
Chief Engineer
San Francisco

Dr. C.B.P.	Ck.
Tr. E.S.R.	Ap.

File No.
2022-46

Fig. 12



SAMPLE NO.	DATE	LEGEND	COORDINATES N	COORDINATES E	ELEVATION	% Moisture	Dry Wt. Lbs/Cu. Ft.
Average for all		Samples	pervious		—	8.0	126.9
"	" "	"	impervious		—	10.1	123.2

SANTA CLARA VALLEY WATER CONSERVATION DISTRICT

ANALYSIS OF
ROLLED FILL MATERIAL
IN STEVENS CREEK DAM

February 1935

Scale as shown

Dr. C.B.P. Ck. E.E.C.
Tr. E.S.R. Ap. [illegible]

Fred. H. Tibbetts
Chief Engineer
San Francisco

File No.
2022-46

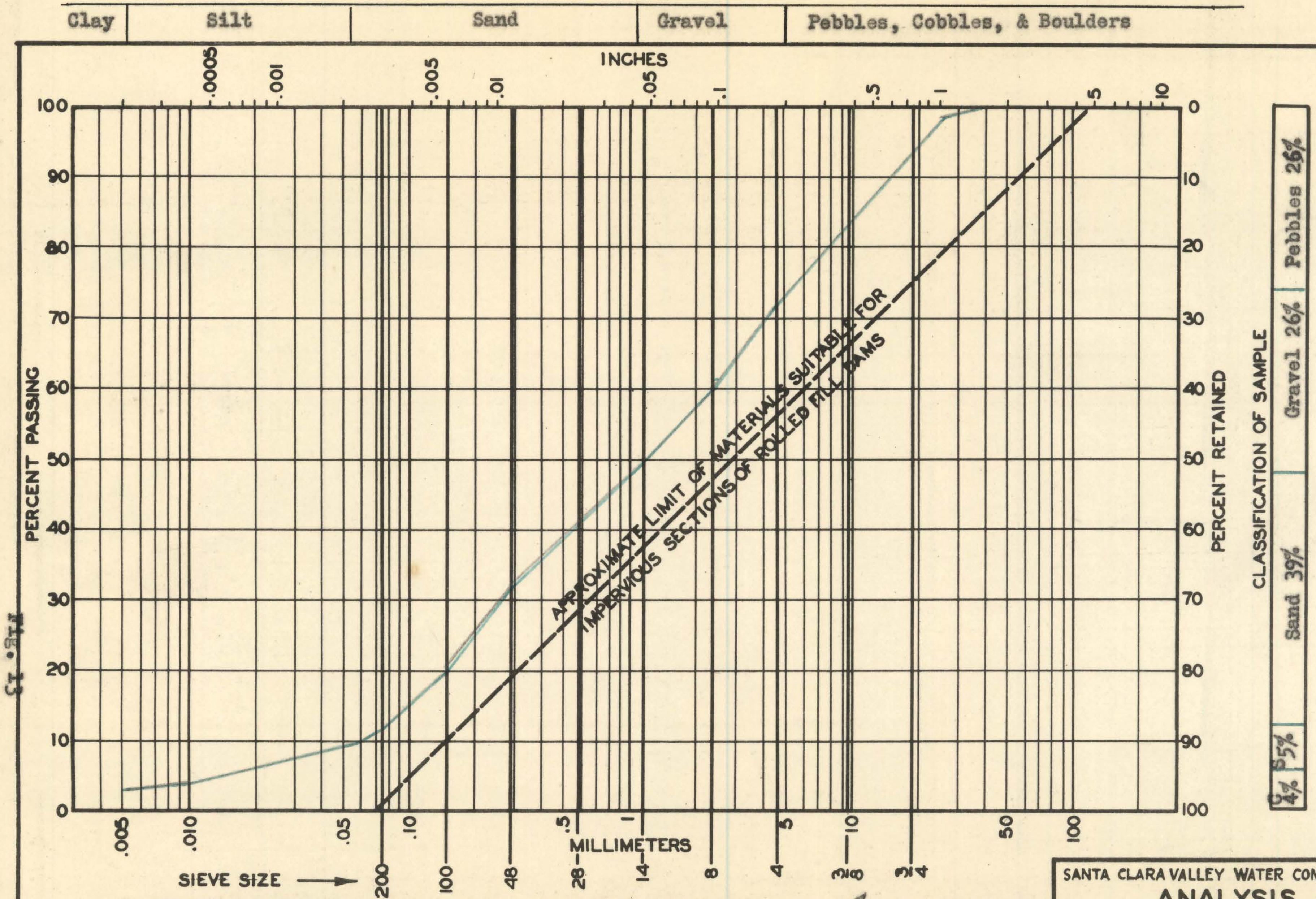


Fig. 13

SAMPLE NO.	DATE	LEGEND	COORDINATES N	COORDINATES E	ELEVATION	% Moisture	Dry Wt. Lbs/cu.ft.
Average for all samples impervious					——	$11^{\underline{0}}$	$124^{\underline{5}}$

SANTA CLARA VALLEY WATER CONSERVATION DISTRICT

ANALYSIS OF ROLLED FILL MATERIAL IN COYOTE DAM

February 1935

Scale as shown

Fred. H. Tibbetts
Chief Engineer
San Francisco

Dr. C.B.P.	Ck. E.S.R.
Tr. E.S.R.	Ap. F.H.T.

File No.
2022-46

ALASKA COMMERCIAL BUILDING
SAN FRANCISCO, CALIF.

FRED H. TIBBETTS
CIVIL ENGINEER

FRED. H. TIBBETTS
HAROLD I. WOOD
E. E. BLACKIE

CONCRETE TEST REPORTS

Project .... Subject .... Date of Report ....

.... Report by ....

Sample No. ....

Date Poured ....

Age at Test ....

Date to be Tested ....

Location of Pour ....

....

Approximate Field Mix

Cement ....

Sand ....

Small Rock ....

Large Rock ....

Slump in Inches ....

Taken by ....

Curing ....

....

Weight / Cu. Ft. ....

Total Load, Pounds ....

Cross-Sectional Area ....

Strength, Lbs. / Sq. In. ....

Tested by

Copies to:

Fig. 14.